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4317 GEOTECHNICAL RPT
(ISSUED 5/6/06)

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Attention: Doug Sarkkinen, PE

SUBJECT: Geotechnical Investigation
Klineline Bridge Replacement
Clark County, Washington

At your request, GRI has completed a geotechnical investigation for the above-referenced bridge project for Clark County, Washington. The bridge site is located on Highway 99 at the crossing of Salmon Creek in Vancouver, Washington. The approximate location of the site is shown on the Vicinity Map, Figure 1. This investigation was conducted to evaluate subsurface conditions at the crossing and develop conclusions and recommendations for design and construction of the proposed replacement structure. The investigation consisted of subsurface explorations, laboratory testing, and engineering studies and analyses. This report describes the work accomplished and provides our conclusions and recommendations for design and construction of the replacement bridge foundations, approaches, and associated retaining walls.

PROJECT DESCRIPTION

The Site Plan, Figure 2, shows the existing creek crossing and surrounding area. The project will consist of replacing the existing four-span bridge with a wider, single-span structure, modifying the existing streambed to limit future down cutting, and providing erosion protection for the existing creek banks. The overall bridge widening will be accomplished on the eastern side of the current roadway alignment. The new bridge will be constructed with pre-cast and pre-stressed concrete girders; the total length and width of the structure will be about 162 and 86 ft, respectively. As currently planned, the new abutments will be supported by drilled shafts with concrete pier caps. The pier caps and other retaining walls will be used to support the ends and sides of the abutment backfill and roadway approach fill.

SITE DESCRIPTION

Topography and Surface Conditions

The creek crossing is located at about elevation 100 ft, on a relatively straight north-to-south-trending section of highway. The creek bed is at about elevation 65 ft. Existing information indicates that the abutment fills are about 10 ft thick on the north side and 20 ft thick on the south side of the existing bridge, and the abutments and intermediate bents of the existing bridge are supported on spread footings. The creek flows westward under the bridge around and between the existing interior bents.

The slope below the north abutment is steep and lies at angles ranging from about 0.7H:1V to nearly vertical; some erosion and raveling are evident on the face of the slope. The slope below the south abutment is somewhat flatter, with the upper portion of the slope about 1.5H:1V and the lower portion at about 3H:1V. Lightly cemented gravel and cobbles are exposed along the lower portion of both slopes. We understand that some scour has occurred at the crossing since the bridge was originally constructed, which has required protecting an intermediate bridge pier foundation with a steel sheet pile cofferdam. In addition, portions of the existing streambed downstream of the bridge were stabilized with grouted riprap, which as resulted in an unnatural waterfall during low stream flow conditions at the downstream edge of the grouted riprap.

Geology

The roadway approaches to the bridge were created by filling over Pleistocene-age flood deposits that mainly consist of unconsolidated, poorly sorted, rounded gravels with a matrix of silt and sand and typically contain cobbles and boulders. The gravels are underlain by cemented sand and gravel with local basaltic lava. At depth, these deposits are underlain by marine mudstone, siltstone, and claystone.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the crossing were evaluated between September 13 and October 26, 2005, with four borings, designated B-1 through B-4. The borings were advanced to depths of 5 to 66 ft at the approximate locations shown on Figure 2. Boring B-2 was abandoned at a depth of about 5 ft on concrete and steel debris. Consequently, boring B-4 was added to the exploration program and drilled on October 26, 2005. Borings B-2 and B-4 were drilled by setting temporary casing to below the stream bottom through holes cored through the existing bridge deck. The field and laboratory testing programs accomplished for this study are described in further detail in Appendix A. Logs of the borings are provided on Figures 1A through 4A. The terms used to describe the soils encountered in the borings are defined in Table 1A.

Soils

For the purpose of discussion, the soils disclosed by the borings have been grouped into the following units based on their physical characteristics and engineering properties. Listed as they were encountered from the ground surface downward, the units are:

1. **PAVEMENT**
2. **Abutment Backfill / Roadway Embankment FILL / Stream FILL**
3. **GRAVEL**
4. **Cemented GRAVEL**

1. PAVEMENT. Borings B-1 and B-3 were drilled through the existing paved roadway surface that exists behind both bridge abutments. Boring B-1, drilled behind the northern abutment, encountered a 16-in.-thick layer of asphaltic-concrete (AC) over a 2-in.-thick layer of crushed rock base (CRB). Boring B-3, drilled behind the southern abutment, encountered a 30-in.-thick layer of AC over a 6-in.-thick layer of CRB.

2. Abutment Backfill / Roadway Embankment FILL. Fill was encountered beneath the existing pavement section behind both bridge abutments. The fill encountered in boring B-1 behind the north abutment consists of fine to coarse, angular to subrounded gravel in a matrix of sandy silt and extends to a depth of about 10 ft. Based on N-values of 37 to 60 blows/ft, the relative density of the gravel fill is considered to be dense to very dense. However, it has been our experience that the Standard Penetration Test may overestimate the relative density of a coarse, granular material.

The fill encountered in boring B-3 behind the south abutment consists of brown, silty sand to sandy silt and extends to a depth of about 20 ft. Based on typical N-values in the range of 3 to 4 blows/ft, the relative density of the silty sand to sandy silt fill is very loose to loose. The natural moisture content of the fine-grained fill ranges from about 20 to 40%.

As previously indicated, boring B-2 was abandoned in concrete and steel debris at a depth of 5 ft below the stream bottom. The steel and concrete debris was capped with about 5 ft of very dense gravel in a matrix of coarse-grained sand.

3. GRAVEL. Gravel was encountered beneath the fill behind both bridge abutments and below the bottom of the creek channel in both the borings made from the bridge deck. The gravel is fine to coarse and subrounded to rounded, may contain cobbles, and generally has a matrix of silt and sand. However, while drilling boring B-3 behind the south abutment, the borehole caved, and circulation of drilling fluid was lost several times between a depth of 27 and 35 ft, indicating there are open-work zones in the gravel and cobbles that lack a fine-grained matrix. N-values of 37 blows/ft to refusal, defined as 50 blows producing less than 6 in. of sampler penetration, indicate the relative density of the gravel and cobbles is dense to very dense. This gravel, which appears not to be cemented, extends to depths of 20 and 35 ft behind the north and south abutments. Boring B-4, drilled in the streambed, was terminated at a depth of 15 ft below the stream bottom in gravels that do not appear to be cemented.

4. Cemented GRAVEL. Borings B-1 and B-3 were both terminated in cemented gravel at a depth of about 65 ft. The cemented gravel is fine to coarse, subrounded to rounded, and contains some coarse, silty sand. Drilling in the cemented gravels progressed very slowly and refusal conditions were consistently encountered, indicating the relative density of the cemented gravel is very dense.

Groundwater

Groundwater levels at the crossing will closely follow creek levels.

CONCLUSIONS AND RECOMMENDATIONS

General

The existing four-span bridge will be replaced with a wider 152-ft-long single-span bridge. The existing bridge is 60 ft wide and will be widened eastward to 86 ft, which will require the eastern side of the southern approach fill to also be widened and retained with a new cast-in-place cantilever retaining wall. Simply supported impact panels will also be provided at both ends of the bridge to accommodate any future settlement behind the abutments. Due to the relatively steep creek banks in close proximity to the location of the planned abutments, particularly along the very steep northern bank, the support of the new abutments on spread footings is not practical. In addition, due to the anticipated difficulty with installing driven piling into the underlying cemented gravels at the site, we understand that drilled shafts are being considered for the support of the new bridge abutments. We also understand the roadway will be closed, and all traffic detoured until the new bridge is completed. In addition, the existing bridge, including the existing piers and grouted riprap in the channel, will be removed.

As part of the bridge replacement, the stream channel beneath and downstream of the bridge will be modified to limit future down cutting. As presently envisioned, the in-water work will consist of placing a series of large stones or boulders in conjunction with installing two sections of driven steel sheet piles to create pools, weirs, and stream-bottom grade-control structures. These treatments with boulders and steel sheet piles will also extend up the sides of the creek banks to serve as erosion control for the banks, and the finished side slopes of the banks will be flattened by placing a buttress of large stones at slopes no steeper that about 2H:1V. The erosion control measures will extend up the sides of the creek banks to the estimated 500-year flood elevation.

The following sections of this report provide our conclusions and recommendations concerning the design and construction of foundation support for the new bridge.

Foundation Support

General. As presently envisioned, each abutment will be supported on a single row of six 36-in.-diameter drilled and cast-in-place, reinforced-concrete shafts. The center-to-center spacing between the shafts along the pier caps will be 15 ft or five pier diameters. The preliminary design loads on each of the shafts from the bridge structure are estimated to be on the order of 575 kips/shaft in compression, 189 kips/shaft laterally (directed transverse to the highway centerline), and 126 kips/shaft laterally (directed longitudinally or parallel to the highway centerline). In the longitudinal direction, an additional lateral load estimated as the lateral earth pressure at rest behind the pier cap also needs to be added to the lateral load from the structure to account for any possible future loss of ground in front of the pier cap. In the transverse direction, the pier tops are assumed to be fixed, and in the longitudinal direction, the pier tops are assumed to be pinned. Our conclusions and recommendations for design and construction of the shafts are provided below.

Axial Capacities. The following recommendations provide estimates of ultimate shaft capacities for use by the bridge designer to aid in the design of the structure. These recommendations have been developed in accordance with FHWA guidelines for the design of drilled shafts. For the north and south abutments, we have estimated that embedded pier lengths of 45 and 50 ft below the bottom of the pier cap will provide an ultimate capacity of about 860 tons. This capacity is based primarily on the skin friction acting along the sides of piers in the underlying gravels with a lesser contribution from the end bearing at the tip of the piers. Settlement and elastic shortening of the shafts under compressive loading is estimated to be less than 1/2 in., depending somewhat on the actual long-term real load transferred to the bottom of the shaft.

We recommend the allowable compressive capacity of the shafts include a minimum factor of safety of 3.0 for long-term dead and live loads based on geotechnical considerations. The allowable compressive capacity of the shafts should include a minimum factor of safety of 2.0 for the total of all loads, including seismic forces, based on geotechnical considerations. Those allowable capacities that are based on Allowable Stress Design (ASD) are not to be compared with factored loads based on Load Resistance Factor Design (LRFD). However, the structural engineer may elect to compare the allowable capacities with the ASD. To compare ultimate capacities to LRFD factored loads, an appropriate resistance factor consistent with AASHTO guidelines needs to be applied to the ultimate capacities to calculate LRFD factored resistances.

Lateral Loading Conditions. For conditions of lateral loading, we anticipate the shafts may be best designed using the computer software "L-Pile Plus Version 5.0." However, as previously indicated, the stream bank geometries and subsurface conditions are somewhat different at the north and south abutment. Ground slope effects at the abutments can be taken into consideration with the input of an appropriate slope angle. At the north abutment, we recommend discounting the lateral restraint that may be provided by any material in front of the base of the pier cap and lying above an angle of 1.5H:1V projected downward in front of the toe of the pier cap to the stream channel. The lateral load of the abutment backfill material retained behind the pier cap should be estimated as an earth pressure at rest based on an equivalent fluid unit weight of 55 pcf, assuming the excavation to complete the pier cap will be open cut and backfilled with granular structural fill. The recommended elevations for the top and bottom of the various soil units at the north abutment are tabulated below.

NORTH ABUTMENT
Recommended Elevations for Soil Unit Input for L-Pile Analysis

	Elevation, ft	
Description	**Top of Unit**	**Bottom of Unit**
Abutment Backfill	Ground surface	Bottom of Cap = 82.6
Gravel	Top of Shaft = 82.6	77.0
Cemented Gravel	77.0	Undetermined

Note: The abutment backfill produces a lateral load against the cap, but does not provide resistance along the shaft.

For the south abutment, we recommend the lateral restraint that may be provided by any material in front of the base of the pier cap and lying above an angle of about 2.5H:1V projected downward in front of the toe of the pier cap to the stream channel. The lateral load of the abutment backfill material retained behind the pier cap should be estimated as an earth pressure at rest based on an equivalent fluid unit weight of 55 pcf assuming the excavation to complete the pier cap will be open cut and backfilled with granular structural fill. The recommended elevations for the top and bottom of the various soil units at the south abutment are tabulated below.

SOUTH ABUTMENT
Recommended Elevations for Soil Unit Input for L-Pile Analysis

	Elevation, ft	
Description	**Top of Unit**	**Bottom of Unit**
Abutment Backfill	Ground surface	Bottom of Pier Cap = 78.7
Sand and Silt Fill	Top of Shaft = 78.7	70.0
Gravel	70.0	55.0
Cemented Gravel	55.0	Undetermined

Note: The abutment backfill produces a lateral load against the cap, but does not provide resistance along the shaft.

The following table provides recommended input parameters for the various soil units for the L-Pile analysis. Since the lateral loading on the shafts is anticipated to be short-term, we recommend the use of undrained soil strength parameters (ϕ = 0°) for the Sand and Silt Fill. For the Cemented Gravel, we anticipate the cementation in the gravel is brittle and may likely fail at very small values of strain. Consequently, we recommend modeling the strength of the cemented gravel as frictional material (c = 0).

SOIL AND ROCK PROPERTIES FOR L-PILE ANALYSIS

		Properties				
Unit	L-Pile Soil Descriptor	K, pci	γ, pci	c, psi	ε_{50}	ϕ
Sand / Silt Fill	Soft Clay	30	0.067	2.78	0.020	0
Gravel	Sand (Reese)	N/A	0.081	0	N/A	35°
Cemented Gravel	Sand (Reese)	N/A	0.081	0	N/A	35°

Note: Where indicated as Not Applicable (N/A), these parameters not input to the program.

The L-pile analysis is applicable for single, isolated pile capacities and for the lateral loading conditions in the transverse direction group effects do not need to be considered since the center-to-center spacing between piles in the direction perpendicular to the load will be greater than three pile diameters. Based on the foregoing, we estimate pier top deflections in the down slope direction of about 1.4 and 1.6 in. for the longitudinal loading conditions for 45 and 50 ft long piers at the north and south abutments, respectively. The deflection, shear, and moment diagrams computed by L-pile for the longitudinal loading at the north and south abutments are provided in Appendix B.

However, for lateral loading conditions in the longitudinal direction group effects need to be considered. To account for the group effects of a six pile group at a five diameter center-to-center spacing in the direction parallel to the load a group factor of 3.75 should be assumed, that is the total capacity of the six pile group is 3.75 times that of a single isolated pile at the same constant value of lateral pile top deflection. Consequently, for the analysis of lateral loading in the transverse direction the load applied at the top of the single pier is the total lateral load on the pier cap divided by 3.75 or about 304 kips. Based on the foregoing, we estimate pier top deflections in the transverse direction of about 0.5 and 1.5 in. for 45 and 50 ft long piers at the north and south abutments, respectively. The deflection, shear, and moment diagrams computed by L-pile for the transverse loading at the north and south abutments are provided in Appendix B, sheets 7B though 12 B, respectively.

Construction Considerations. In general, we recommend that all drilled shaft construction be accomplished in conformance with the Washington State Department of Transportation (WSDOT) specifications. During construction, it should be anticipated that temporary casing will be necessary to prevent caving conditions. In addition, we anticipate that it may not be practical to dewater inside the casing; consequently, the drilled shaft contractor should be prepared to place the concrete by tremie methods. If tremie methods are used, the contractor must maintain a minimum of 5 ft of concrete head above the bottom of the temporary steel casing as the casing is removed. We further recommend that all shafts be completed in the shortest possible sequence. In addition, the methods of construction can significantly influence the long-term behavior of the shaft under compressive loading, such as any loose or soft materials left at the bottom of the shaft after drilling; consequently, we recommend the bottom of the shafts be well cleaned prior to casting the shafts. In this regard, we recommend that there be no more than 2 in. of loose material at the bottom of the shaft just prior to placing the concrete.

Crosshole Sonic Logging (CSL) tubes should be incorporated into the design of the reinforcing cages for the shafts. Installing the CSL tubes with the reinforcing cages provides a means of nondestructive testing to verify the homogeneity and integrity of the concrete cast in the shafts and can be used to identify anomalies, such as voids or soil intrusions within a shaft. The tubes may be installed with the reinforcing cages and used to evaluate the integrity of individual shafts. We recommend that a minimum of four equally spaced CSL tubes be provided for each of the shafts. Rock coring techniques may also be used in conjunction with CSL testing to evaluate the integrity of any shafts, if the CSL testing is not conclusive.

To observe compliance with the intent of our recommendations and design concepts, and the project plans and specifications, we are of the opinion that all construction operations dealing with drilled shafts should be observed by a GRI representative on a full-time basis.

Retaining Walls

General. We understand the pier caps will be designed to retain the bridge approach fill, and a conventional cast-in-place, cantilever retaining wall will also be required to retain the eastern side of the southern approach embankment to accommodate the highway and bridge widening. Design lateral earth pressures behind retaining walls depend on the backfill geometry and the drainage condition behind the wall and the ability of the wall to yield by either translation or rotation away from the backfill. The two possible conditions regarding the ability of a wall to yield include yielding and non-yielding walls. An example of a yielding wall is a conventional cantilevered retaining wall supported on a spread footing which yields by sliding or rotating about its base. An example of a non-yielding wall is a wall that is laterally supported at its top and base, such as the pier caps that will be used to retain the approach backfill. Yielding and non-yielding walls can be designed on the basis of a hydrostatic pressure based on an equivalent fluid unit weight 35 and 55 pcf, respectively. These pressures assume that the wall supports a horizontal backfill that is fully drained.

Surcharge Loads. Surcharge loads from traffic do not need to be added to the earth pressure acting behind the pier caps if simply supported impact panels are provided at each approach. Surcharge loads due to traffic should be added to the loads behind the cantilevered retaining wall and may be estimated using the guidelines shown on Figure 3; however, we recommend a minimum uniform surcharge of 200 psf to account for construction equipment and traffic.

Seismic loading conditions can also result in additional earth pressure behind retaining walls. To account for seismic loading, the recommended lateral earth pressures should be increased by 40%. The resultant of the additional seismic force can be assumed to act at a distance of 0.6H measured up from the base of the wall, where H equals the overall height of the wall. However, the extent to which these seismic forces are taken into consideration in the design of walls for highway projects typically depends on the anticipated mode and consequences of failure. For example, for conventional cantilever walls constructed in highly active seismic areas, such as portions of California, the performance of these wall types not designed for seismic loads has been good since the normal mode of wall failure during a seismic event has been by sliding and not catastrophic collapse, and the usual consequence of sliding generally includes visual distortion of the wall and/or subsidence in the wall backfill area.

Wall Backfill. The backfill behind retaining walls should be fully drained. A minimum 2-ft-wide vertical drainage blanket should be placed behind the back of the wall. The drainage blanket material should consist of a drain rock product that contains less than about 2% passing the No. 200 sieve (washed analysis) and meets the gradation requirements of WSDOT specifications for gravel backfill for drains. Weep holes or a perforated drain pipe should be provided near the bottom of the drainage blanket near the base of the wall. Overcompaction of the backfill behind walls should be avoided. In this regard, we recommend compacting the backfill to about 95% of the maximum dry density as determined by AASHTO T-99.

Foundation Support. We anticipate the spread footing for the retaining wall along the eastern side of the southern bridge approach will likely be established in silt and sand embankment fill. The spread footings should be underlain by a minimum 2-ft-thick layer of well-graded crushed rock, such as 4-in.-minus crushed rock, capped with a 4-in.-thick leveling course of 3/4-in.-minus crushed rock. The crushed rock should be compacted until either well-keyed or to a minimum of 95% of the maximum dry density as determined by AASHTO T-99. Excavations for footings should be made with a hydraulic excavator equipped with a smooth-edge bucket, and all footing excavations should be observed by a qualified geotechnical engineer. Any soft, unsuitable soils encountered after completing the recommended 2 ft of overexcavation should be overexcavated to firm, undisturbed ground and backfilled with additional granular structural fill. We anticipate that local areas of the footing subgrade will require overexcavation and backfilling with structural fill in addition to the minimum 2 ft recommended beneath the footings. In our opinion, spread footings prepared as above can be designed to impose an allowable bearing value of up to 3,000 psf, and the resultant of the pressure should lie within the middle third of the base of the wall. This value applies to the total of dead load plus frequently and/or permanently applied live loads and can be increased by one-third for the total of all loads. The allowable bearing value includes a factor of safety of at least 3 based on a bearing capacity-type failure. Spread footings should have a minimum width of at least 3 ft.

We estimate the settlement of spread footings established in accordance with the above recommendations and supporting up to about 12 ft of retained fill will be on the order of 1 in. and will occur rapidly as the loads are applied. The front face of the wall should be battered back to allow for any yielding of the wall by rotation.

In our opinion, since cast-in-place walls are somewhat more sensitive to settlement than other available types of wall systems, such as mechanically stabilized earth (MSE) walls, we recommend placing the majority of the new approach fills early in the construction schedule to allow for settlement prior to completing finished surfaces such as paving and sidewalks. In this regard, we recommend that after the fill has been placed, a series of monitoring hubs be established and surveyed at least twice a week until the rate of settlement has decreased to relatively small amounts. The settlement monitoring data should be provided to GRI for evaluation as the information is collected. In this regard, we anticipate the majority of the settlement will occur relatively quickly during the course of construction of the fills. However, in the deeper fill areas, settlement may continue for up to several weeks after construction of the structural fills.

Footings should be provided with at least 3 ft of embedment. Horizontal forces can be resisted partially or completely by frictional forces developed between the base of the footing and the subgrade soil, and we recommend an ultimate value of 0.35 for the coefficient of friction. If additional lateral resistance is required, passive earth resistance against embedded footings can be computed on the basis of an equivalent fluid having a unit weight of 230 pcf; however, this value assumes the ground line in front of the footing is horizontal for a distance equal to at least three times the footing embedment. In the event that the finished ground line in front of the footing may slope downward, the following table provides a summary of reduced passive earth pressure values in terms of equivalent fluid unit weights that we recommend using in the design.

	Passive Pressure Equivalent Fluid Unit Weight	
Slope Angle	**Saturated Condition**	**Fully drained Condition**
Horizontal	230 pcf	500 pcf
3H:1V	85 pcf	190 pcf
2H:1V	50 pcf	115 pcf

Notes:

(1) Above passive pressures include a factor of safety of 2.0 to limit movement at the toe of the wall.

(2) Drained conditions would be applicable to granular structural fill meeting the requirements of free-draining fill material, i.e., clean, coarse, crushed rock containing less than 2% passing the No. 200 sieve (washed analyses) and for wall foundations established at an elevation above high river level.

(3) Saturated conditions pertain to structural fill not meeting the requirements of clean granular fill that may become saturated due to intense, prolonged precipitation, or for wall foundations established at an elevation below flood levels.

Earthwork

The silty soils that mantle some of the project area are sensitive to moisture and have a moisture content that is above optimum during the majority of the year. Water levels at the site are expected to parallel creek levels; however, perched water levels may exist during the wetter winter and spring months. When the on-site silty soils are in excess of their optimum moisture content, they soften and become unsuitable when remolded by construction traffic. Placement of granular structural fill may be necessary to provide access during foundation construction and for any modifications to the approach embankments and the stream channel improvements.

The ground surface within fill, pavement, or shallow foundation areas should be stripped of vegetation, surface organics, and loose surface soils. The required depth of stripping will depend on vegetation types. In grassy areas, a stripping depth of 6 to 8 in. is usually adequate to remove the majority of the rooted zone. In other areas, deeper grubbing will be required to remove brush and tree roots. Due to their high organic content, strippings should be disposed of off-site. Following stripping or excavation to subgrade levels, the exposed soils should be evaluated by a geotechnical engineer to detect any soft areas. Soft areas should be overexcavated and replaced with structural fill. During and following stripping and excavation, the contractor must use care to protect the subgrade from disturbance by construction traffic.

All excavations in the fine-grained soils that mantle the site should be made using large hydraulic excavators (trackhoes) equipped with smooth cutting edges, in lieu of other equipment, such as bulldozers, to prevent softening of the subgrade soils. Also, we recommend that all construction traffic in areas with silty subgrade soils be limited to movement on granular work pads or haul roads to avoid softening the exposed fine-grained soils. A minimum of 12 to 24 in. of relatively clean, angular rock placed over a geotextile fabric is generally required to support construction traffic and protect the subgrade. The use of a fabric serves to reduce maintenance of the section during construction.

If the subgrade is disturbed by construction traffic, soft or loose areas should be overexcavated to firm soil and backfilled with structural fill materials consisting of imported, clean granular material with a maximum size of less than 8 in. and not more than about 5% passing the No. 200 sieve (washed analysis). The granular structural fill materials should be compacted with vibratory equipment to 95% of the maximum dry density as determined by AASHTO T-99 or until well keyed. Lift thicknesses should be proportionate to the type of compaction equipment used. For example, the first lift of granular fill placed over a fine-grained subgrade should be about 18 to 24 in. thick, and subsequent lifts should be about 12 in. thick when using medium- to heavy-weight vibratory rollers. Lift thicknesses should be limited to less than 8 in. when using hand-operated vibratory plates. In our opinion, all structural fills should extend a minimum of 5 ft beyond the edge of the new roadway and/or adjacent improvements.

The use of fine-grained soils for structural fill will be limited to prolonged periods of dry weather when these soils can be moisture conditioned to near optimum for proper placement and compaction. Fine-grained fill materials should be compacted with a segmented-pad roller to 95% of the maximum dry density as determined by AASHTO T-99. Appropriate lift thicknesses are generally on the order of 6 to 8 in.

Excavation

The method of excavation and support of excavation sidewalls are the responsibility of the contractor and subject to applicable local, state, and federal safety regulations regarding excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for use by the owner and engineer and should not be interpreted to mean that GRI is assuming responsibility for the contractor's actions or site safety.

Several of the improvements will require excavations on the order of 10 ft below existing site grades. Our experience indicates an open cut is the most practical approach to accomplishing these excavations. The borings disclosed that the excavations at the new bridge abutments will encounter a variety of materials, including dense gravels at the north abutment and very loose to loose, silty and sandy materials at the south abutment. While it is likely these materials can be excavated economically with large hydraulic excavators, it should be anticipated that progress may be slowed if the excavations encounter the very dense, cemented gravels present in the area of the north abutment.

Based on our evaluation of the materials disclosed in the borings, the short-term stability of temporary cut slopes of 1H:1V should be adequate. However, this assumes the slopes are dewatered; surface drainage is controlled so that runoff does not flow over the top of the slope and into the excavation; and surcharge loads due to construction traffic, material laydown, excavation spoils, etc., are not allowed within 5 ft of the top of the cut. In this regard, we recommend the use of fencing or barricades along the top of the cut to prevent this area from being subjected to any significant surcharge loads.

It must be emphasized that following the above recommendations will not guarantee that failure of the temporary cut slopes will not occur; however, the recommendations should reduce the risk of a major slope failure to an acceptable level. It should be realized, however, that blocks of ground and/or localized slumps in the excavation slopes may tend to move into the excavation during the construction. In our opinion, this is most likely to result if seepage occurs on the cut slopes and/or if the cut slope exposes relatively thick deposits of poorly graded granular soils, which have the potential to run into the excavation.

Erosion Protection

The existing creek banks are relatively unprotected and subject to erosion during high water events. In particular, the northern creek bank in the area of the new bridge is relatively steep and there are indications of ongoing erosion. In conjunction with the in-water improvements to limit future down cutting in the area of the new bridge, we recommend protecting the existing creek banks against future erosion. In this regard, we understand the average stream velocities could approach 10 ft/sec during high water and/or flood events. Based on these relatively high velocities, we recommend the slope protection consist of heavy loose or hand-placed riprap meeting the requirements of WSDOT. In this regard, the actual size of the stones for the hand-placed riprap slope protection should be made on the basis of the anticipated floodwater velocities predicted by a hydraulic analysis.

In conjunction with the addition, we recommend the use of a filter blanket with a minimum thickness of 12 in. measured perpendicular to the slope. The filter blanket material can consist of quarry spalls meeting the requirements of WSDOT. The riprap slope protection should extend up to the estimated 500-year flood elevation and a minimum horizontal distance of 25 ft upstream and downstream of the new bridge. In conjunction with the placement of the slope protection, we understand the surface of the riprap will likely be dressed with woody debris and/or filled in with finer material to facilitate the grow of vegetation to enhance the stream habitat.

We recommend that all new cut and fill slopes, including riprap slope protection, be constructed no steeper than 2H:1V. All new fill prisms should be keyed into existing slopes with a series of horizontal benches. We further recommend that all newly exposed soils above the estimated 500-year flood elevation be protected against erosion. Common methods of controlling erosion include seeding and mulching or applying a protective blanket of granular material over the exposed soil. If seeding and mulching is used to control surface erosion, the slope areas should be lightly scarified prior to applying the seeds and mulch. The scarifying helps the seeds adhere to the slope, giving them a better chance to grow.

Construction Considerations

Based on the subsurface explorations, it should be anticipated that relatively difficult driving conditions will be encountered during installation of the steel sheet piles. In this regard, it should be anticipated that installation of the sheet piles will require a relatively large impact hammer, such as a Delmag D30 with a rated energy up to 54,250 ft-lbs, and relatively hard driving to install the piles to the design tip elevation. In our opinion, a vibratory hammer is not suitable for the subsurface conditions at this site. We also recommend the sheet piles consist of the heaviest steel section commonly available to facilitate installation into the dense sand and gravel deposit, which likely contains cobbles. In addition, it may be necessary to remove some of the near-surface debris in the creek prior to driving the steel sheet piles. If encountered, these obstructions could likely be removed with large hydraulic excavators that will likely be used to remove the existing grouted riprap and place the new stone grade control structures and riprap slope protection.

In our opinion, the contractor's management of groundwater within the excavations required for the stream channel modifications will be a major consideration. In this regard, we understand that construction of the new grade control structures will require excavations up to 10 ft below the existing stream channel. We understand the current plan is to perform this work during the summer when stream flows are the lowest. In addition, we understand that a temporary bypass will be used to divert the stream flows around the work area. This bypass will require the contractor to construct a dike upstream of the work area to channel the stream flows into a conduit(s) that will discharge downstream of the work area.

We anticipate that groundwater encountered within excavations will result in seepage, possibly running soil conditions, and unstable excavation sides and bottom. These conditions will require dewatering of excavations. The amount of groundwater observed in the excavations will depend on the overall permeability of materials encountered and the depth of the excavation below the surface of the groundwater. Some loss of drilling fluid was noted in the subsurface explorations, which indicates that open-work zones exist within the gravel deposits. These types of deposits have the potential to produce large groundwater flows that will likely require the use of dewatering wells that extend below the depth of excavation. As discussed above, the short-term stability of temporary cut slopes of 1H:1V should be adequate; however, this assumes the slopes are dewatered.

The contractor's dewatering system should be designed to maintain groundwater levels to at least 1 ft below the bottom of the excavation. In our opinion, the dewatering system should be designed by a professional engineer experienced in the design of dewatering systems and registered in Washington.

Pavement Design

Borings B-1 and B-3 were drilled through the existing roadway surface that exists behind both bridge abutments. Both borings encountered a considerable thickness of asphaltic concrete (AC). Boring B-1, drilled behind the northern abutment, encountered a 16-in.-thick layer of AC over a 2-in.-thick layer of crushed rock base (CRB). Boring B-3, drilled behind the southern abutment, encountered a 30-in.-thick layer of AC over a 6-in.-thick layer of CRB.

We understand that Clark County Roadway Standards provide for typical pavement sections based on the classification of the roadway and the AASHTO soil type of the subgrade. We recommend the thickness of the new pavement sections be conservatively based on the worst-case subgrade condition behind the abutments and the same section be provided at both approaches. In this regard, the worst-case subgrade condition should be assumed to be a silt fill, and the subgrade should be classified as soil type A-5 in accordance with the AASHTO soil classification system. To estimate the traffic-carrying capacity of the typical county section in accordance with AASHTO design guidelines, we recommend assuming a California Bearing Ratio (CBR) of 4.0% and a corresponding resilient modulus of 6,000 psi to characterize subgrade support for a soil type A-5 subgrade. In our opinion, appropriate layer coefficients for the AC and crushed rock base may be assumed to be 0.44 and 0.14, respectively. We would be glad to review the adequacy of the County Standard section for a 20-year design life once the roadway classification is known and if 20-year traffic estimates can be provided. We recommend that all roadway materials and workmanship conform to the applicable standards of Clark County and WSDOT; however, we further recommend placing a geotextile fabric between any fine-grained subgrade and new granular base course to serve as separation membrane between the base and subgrade.

Seismic Considerations

We understand the new bridge structure will be designed to withstand forces from a 500-year return interval seismic event (probability of exceedance of 10% in 50 years). The WSDOT Geotechnical Manual, M46-03, recommends an earthquake-induced peak bedrock acceleration of at least 0.22 g for a 500-year return interval seismic event.

The manual also states the soil classification and response modification factors should be taken from AASHTO Bridge Design Specifications. The AASHTO guidelines for seismic design of highway bridges recommend the effects of site conditions on bridge response be determined from a site coefficient, S, based on soil profile types defined as follows:

> SOIL PROFILE TYPE I is a profile with either: (1) Rock of any characteristic, either shalelike or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/s, or by other appropriate means of classification); or (2) stiff soil conditions where the soil depth is less than 200 ft and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
>
> SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
>
> SOIL PROFILE TYPE III is a profile with soft to medium-stiff clays and sands, characterized by 30 ft or more of soft to medium stiff clays with or without intervening layers of sand or other cohesionless soils.
>
> SOIL PROFILE TYPE IV is a profile with soft clays or silts greater than 40 ft in depth. These materials may be characterized by a shear wave velocity less than 500 ft/s and might include loose natural deposits or synthetic, non-engineered fill.

AASHTO also states, "In locations where soil properties are not known in sufficient detail to determine the soil profile type, or where the profile does not fit any of the four types, the site coefficient for Soil Profile Type II shall be used."

Based on the borings made at the site, the soil mantling the location of the north abutment consists of about 10 ft of fill comprised of dense, angular gravel in a matrix of sandy silt to silty sand, underlain by dense to very dense gravel. At the south abutment, the soils mantling the site consist of about 20 ft of very loose to loose, fine-grained fill underlain by medium dense to very dense gravel. Consequently, we recommend using Soil Profile Type II in the seismic analysis for the new bridge.

In terms of other potential seismic-related hazards in the area, we anticipate the risk of liquefaction, lateral spreading, settlement, or ground subsidence is low. Based on the location of known and mapped faults in the area, we anticipate the potential for fault rupture or displacement at the site is absent, unless occurring on a previously unknown or unmapped fault. Based on the location and elevation of the area, the risk of damage by tsunamis and/or seiches at the site is absent.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that the installation of the drilled piers should be observed full-time by a GRI representative. In addition, other aspects of the new construction such as the bearing surfaces beneath retaining wall foundations and any fill or backfill placement and compaction should also be observed on an as-called-for basis by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid the design team in the design and construction of the Klineline Bridge replacement over Salmon Creek on WA Highway 99 in Clark County, Washington. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the bridge structure and associated highway improvements. In the event that any changes in the design or location of the alignment or structure, as outlined in this report, are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on the Site Plan, Figure 2, and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between boring locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

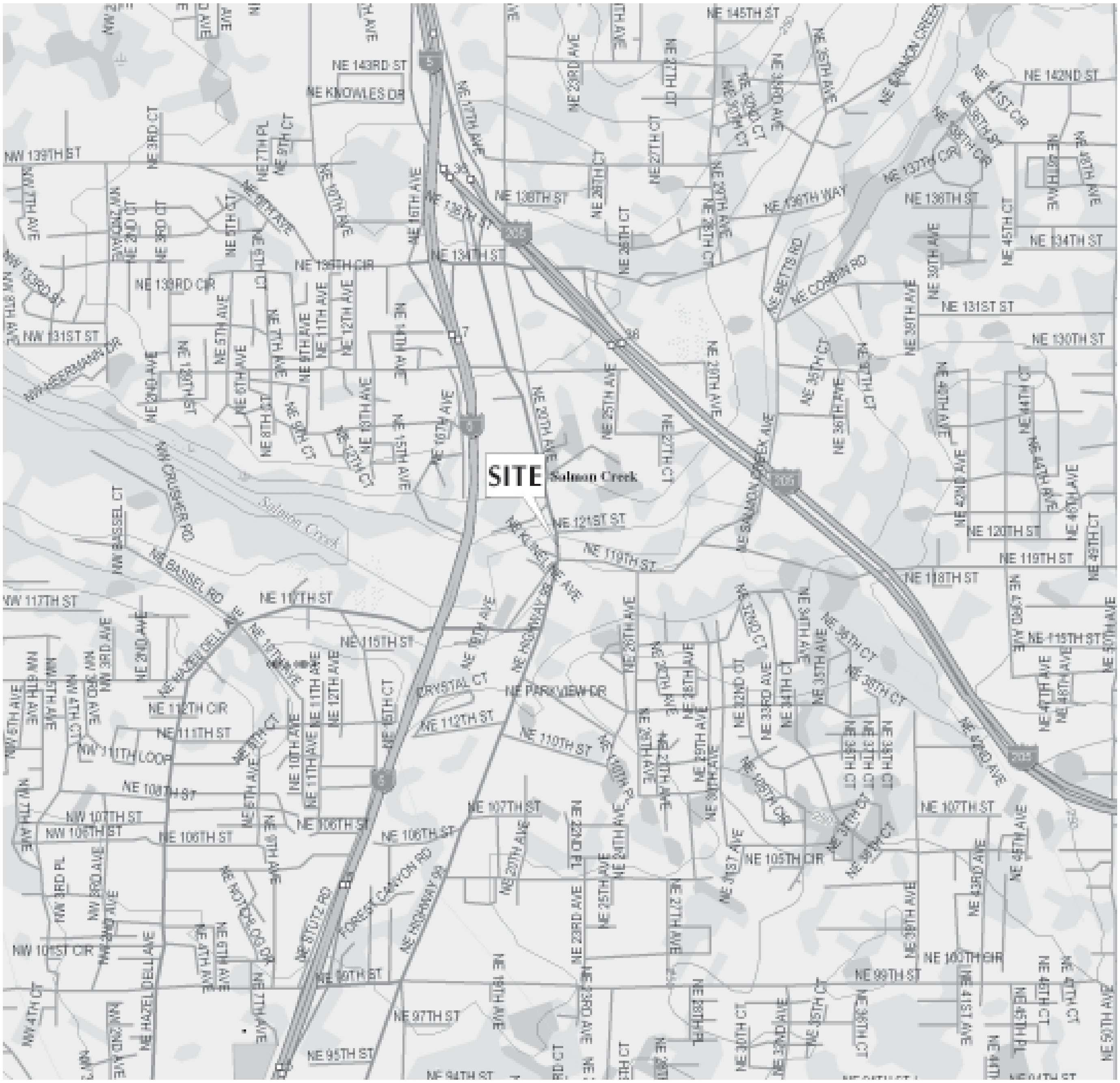
Submitted for GRI,

H. STANLEY KELSAY
STATE OF WASHINGTON
20,498
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 7/25/06

H. Stanley Kelsay, PE, GE	Philip L. Wurst, PE, GE	Tova R. Peltz, PE, RG
Principal	Senior Engineer	Project Engineer/Geologist

References

Gannett, M.W. and Caldwell, R.R., 1998, Geologic framework of the Willamette Lowland aquifer system, Oregon and Washington: U.S. Geological Survey Professional Paper 1424-A.



DELORME 3-D TOPOQUADS, WASHINGTON
VANCOUVER, WASH. (1db) 2004

North

0 1/2 1 MILE

GRI

KRAMER-GEHLEN & ASSOCIATES, INC.
KLINELINE BRIDGE REPLACEMENT

VICINITY MAP

DEC. 2005 JOB NO. 4317 FIG. 1

BORING MADE BY GRI
(SEPTEMBER 13 - OCTOBER 26, 2005)

SITE PLAN FROM FILE BY MINISTER-GLAESER SURVEYING INC., DATED AUGUST 31, 2005

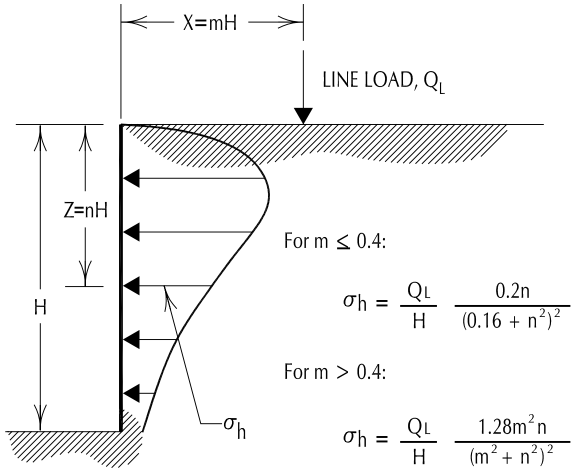
North

0 80 160 FT

GRI KRAMER-GEHLEN & ASSOCIATES, INC.
KLINELINE BRIDGE REPLACEMENT

SITE PLAN

DEC. 2005 JOB NO. 4317 FIG. 2



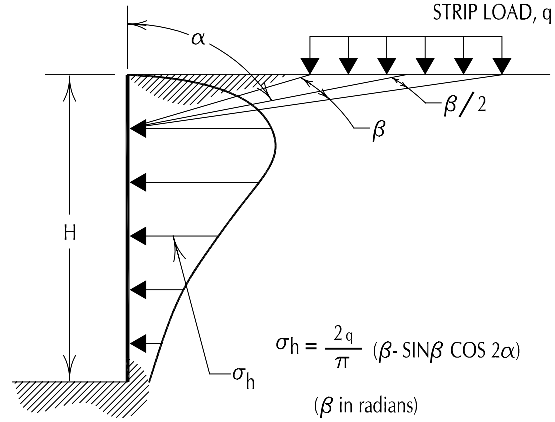
For m ≤ 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For m > 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

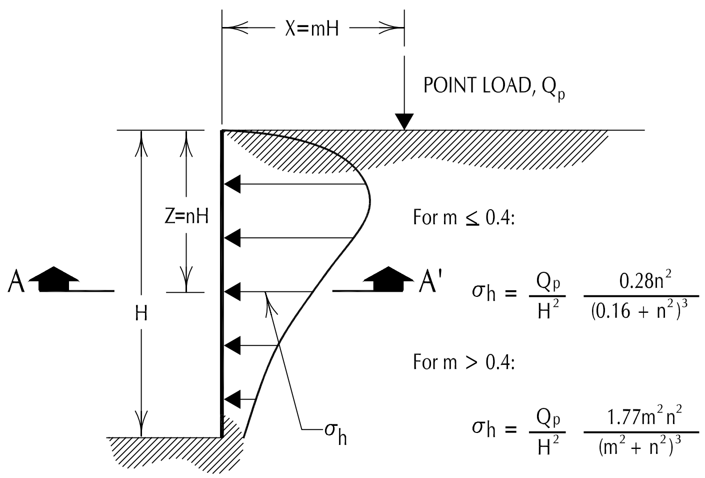
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL

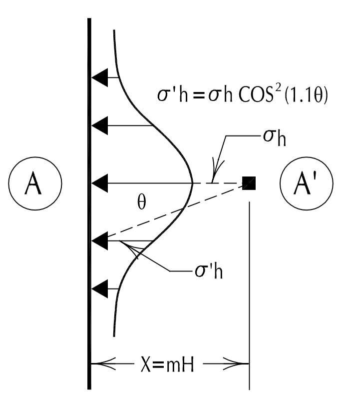


For m ≤ 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For m > 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI KRAMER-GEHLEN & ASSOCIATES, INC.
KLINELINE BRIDGE REPLACEMENT

SURCHARGE-INDUCED LATERAL PRESSURE

DEC. 2005 | JOB NO. 4317 | FIG. 3

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were explored with four borings, designated B-1 through B-4. Borings B-2 and B-4 were made by setting temporary casing to below the stream bottom through holes cored through the existing bridge deck. Borings B-1 through B-3 were drilled between September 13 and 15, 2005. Boring B-2 was abandoned on concrete and steel debris at a depth of 5 ft. Consequently, boring B-4 was added to the exploration program and drilled on October 26, 2005. The approximate locations of the borings are shown on the Site Plan, Figure 2. The borings were drilled using a truck-mounted Mobile B-57 drill rig provided and operated by Subsurface Technologies of Banks, Oregon. The total depth of the borings ranged from about 5 to 66 ft. A geotechnical engineer from GRI maintained a detailed log of the materials and conditions encountered in each boring and collected representative soil samples.

Mud-rotary tools and drilling methods were used to advance the borings. Disturbed and undisturbed soil samples were typically recovered from the borings at 2.5- to 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. The Standard Penetration Test was conducted at the time of sampling. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140 lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the degree of compactness of granular soils, such as sand, and the degree of softness or stiffness of cohesive soils, such as silt. All of the split-spoon samples were saved in airtight jars and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 4A. Each log provides a descriptive summary of the various types of materials encountered and notes the depth where the material and/or characteristics of the material change. The terms used to describe the soil are defined in Tables 1A. To the left of the descriptive summary, a graphic log indicates the general soil or rock types encountered in the borings. To the right of the descriptive summaries the depths and types of samples are indicated. N-values and moisture contents are also shown, where applicable.

LABORATORY TESTING

General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. At the time of classification, the natural moisture content of each soil sample was determined.

Natural Moisture Content

The natural moisture content of the soil samples was determined in conformance with ASTM D 2216. The test results are shown on Figures 1A through 4A.

Washed Sieve Analysis

Washed sieve analyses were performed for selected soil samples obtained from borings B-1 and B-3 to evaluate the portion of material passing the No. 200 sieve. These tests were performed to assist in material classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven dried and weighed. The percentage of material that passed the No. 200 sieve is then calculated. Test results are summarized below.

Boring	Sample	Depth, ft	% Passing the No. 200 Sieve	Soil Type
B-1	S-4	2.5	7	SAND; fine grained, trace silt
B-3	S-3	7.5	51	Sandy SILT

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials that exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

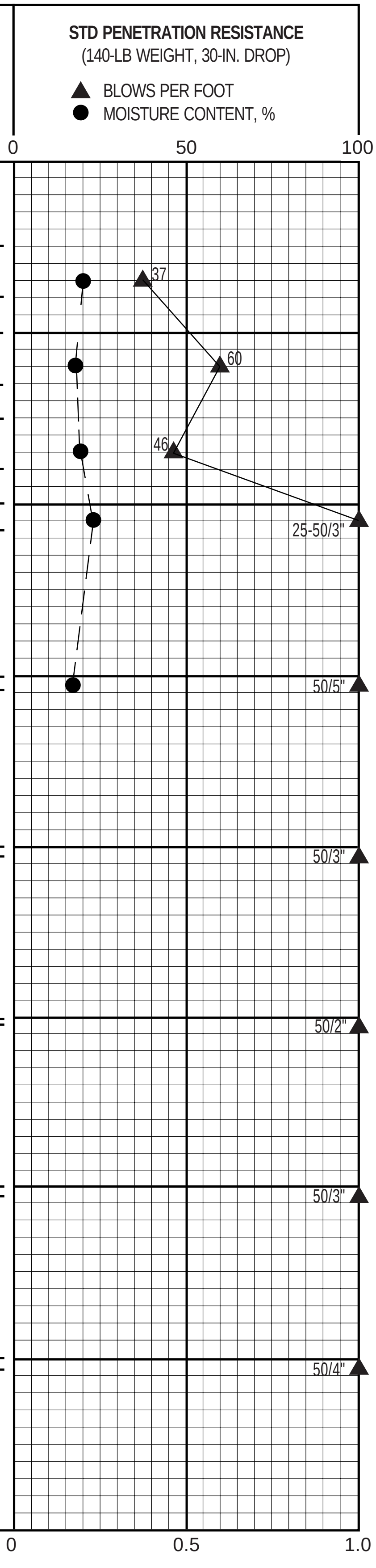
Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

DEPTH, FT	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 97 ft (±)	DEPTH, FT	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP)
0	Asphaltic-concrete PAVEMENT (16 in.) over crushed rock BASE COURSE (2 in.)			
	FILL: Dense to very dense GRAVEL; fine to coarse, angular to subrounded, in matrix of sandy silt	1.5	S-1	37
5			S-2	60
			S-3	46
10	Very dense, gray GRAVEL; fine to coarse, subrounded to rounded, in matrix of silt and sand	10.0	S-4	25-50/3"
15			S-5	50/5"
20	---------cemented; very slow drilling below 20 ft		S-6	50/3"
25			S-7	50/2"
30			S-8	50/3"
35			S-9	50/4"
40				



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

BORING B-1

DEC. 2005 JOB. NO. 4317 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
		SURFACE ELEVATION 97 ft (±)				0 50 100
40		Very dense, gray GRAVEL; fine to coarse, subrounded to rounded, scattered cobbles, in matrix of silt and sand, cemented, very slow drilling				
					S-10	50/3"
45					S-11	50/4"
50					S-12	50/4"
55					S-13	50/3"
60					S-14	50/4"
65			65.3		S-15	50/3"
		(9/13/2005)				
70						
75						
80						0 0.5 1.0 (TONS PER FT ²)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▌ NX CORE RUN

SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

BORING B-1 (cont.)

DEC. 2005 JOB. NO. 4317 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 62 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, % 0 50 100
		Very dense GRAVEL; fine to coarse, in matrix of coarse-grained sand ---------encountered buried steel and concrete debris at 5 ft			S-1 I	50/3" ▲
5		Practical refusal at 5 ft (9/15/2005)	5.0			
10						
15						
20						
25						
30						
35						
40						0 0.5 1.0 (TONS PER FT ^{2})

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

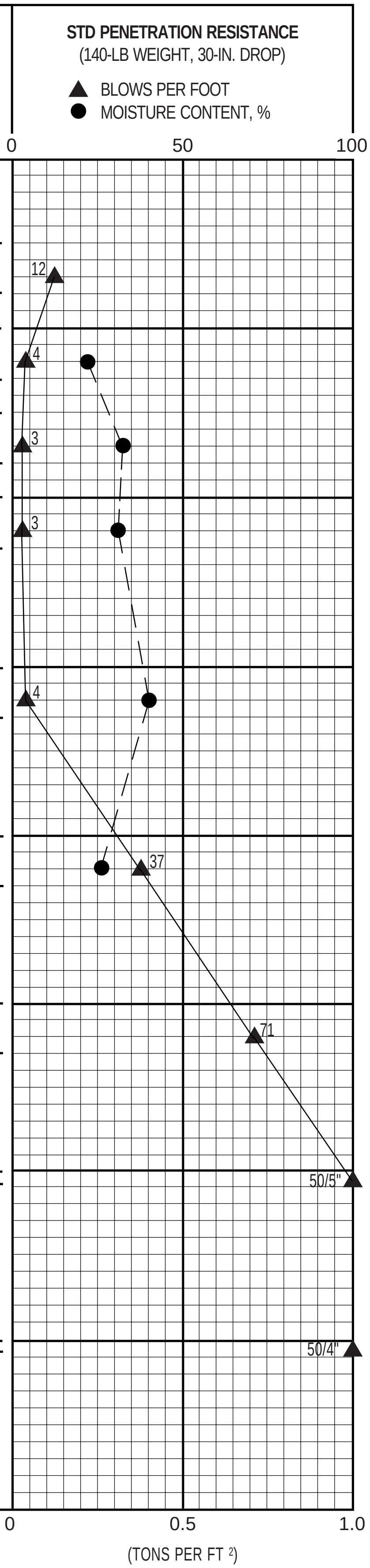
GRI

BORING B-2

DEC. 2005 JOB. NO. 4317 FIG. 2A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT
	SURFACE ELEVATION 90 ft (±)	
0 – 3.0	Asphaltic-concrete PAVEMENT (30 in.) over crushed rock BASE COURSE (6 in.)	3.0
3.0 – 20.0	FILL: Very loose to loose, brown, silty SAND to sandy SILT; fine-grained sand	20.0
20.0 – 27.0	Dense to very dense, gray GRAVEL; fine to coarse, subrounded to rounded, some silty sand to sand	27.0
27.0 – 35.0	Dense to very dense GRAVEL and COBBLES; rounded to subrounded, in matrix of loose, fine-grained sand Driller's Note: Loose matrix, loss of drilling fluid circulation, sidewall caving.	35.0
35.0 – 40	Very dense GRAVEL and COBBLES; coarse, subrounded to rounded, cemented, very slow drilling	

SAMPLES	BLOWS PER FOOT
S-1 *	12
S-2	4
S-3	3
S-4	3
S-5	4
S-6	37
S-7	71
S-8	50/5"
S-9	50/4"



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

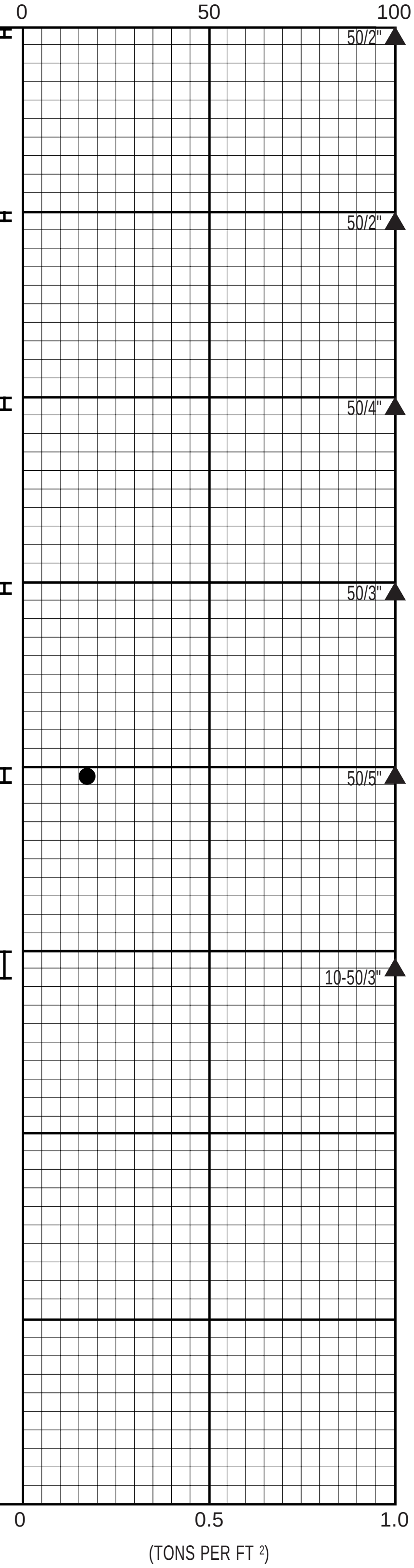
BORING B-3

DEC. 2005 JOB. NO. 4317 FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 90 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40		Very dense GRAVEL and COBBLES; fine to coarse, subrounded to rounded, some coarse-grained sand, cemented, very slow drilling			S-10
45					S-11
50					S-12
55					S-13
60					S-14
65			65.8		S-15
		(9/14/2005)			
70					
75					
80					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

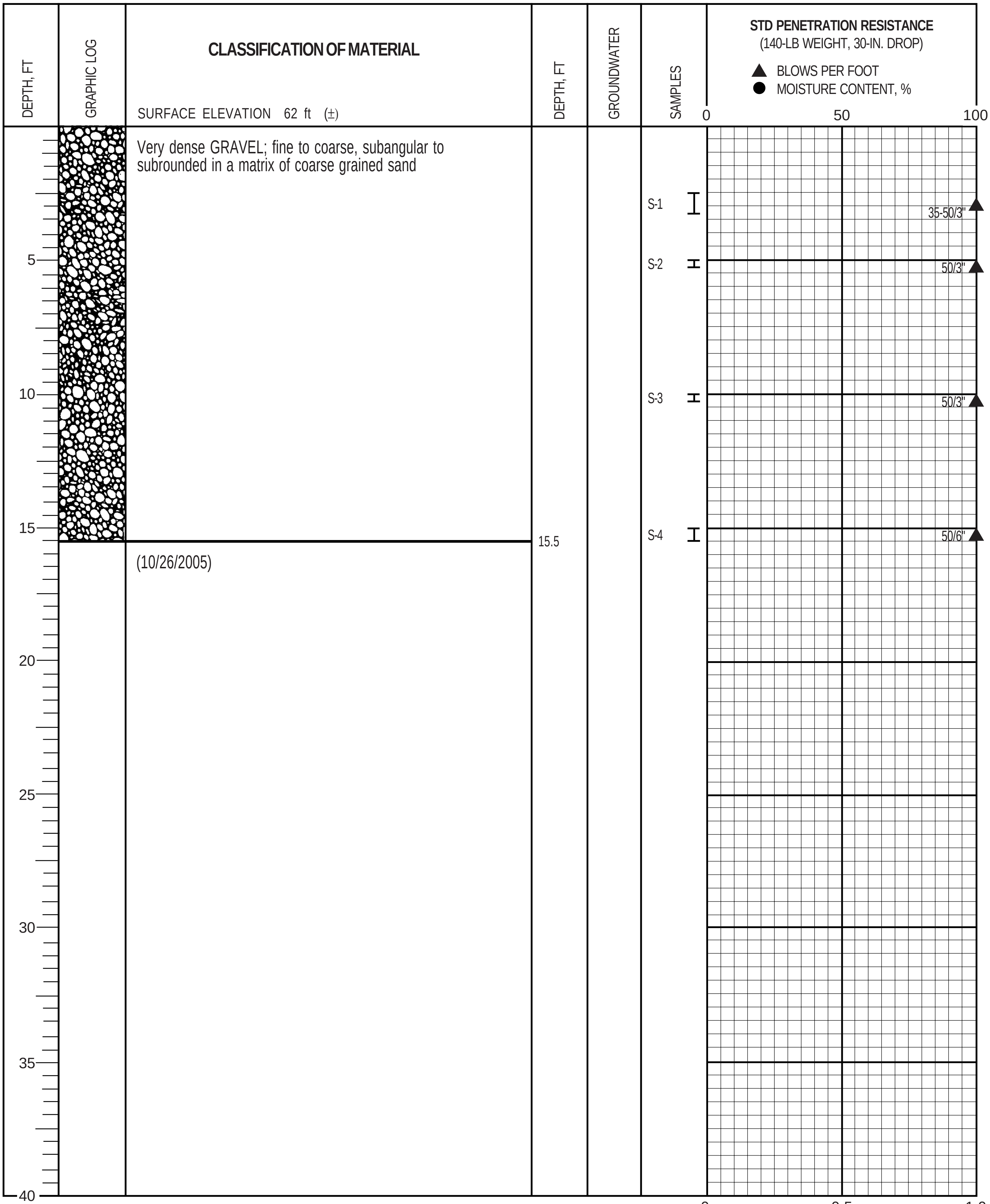
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-3 (cont.)

DEC. 2005 JOB. NO. 4317 FIG. 3A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

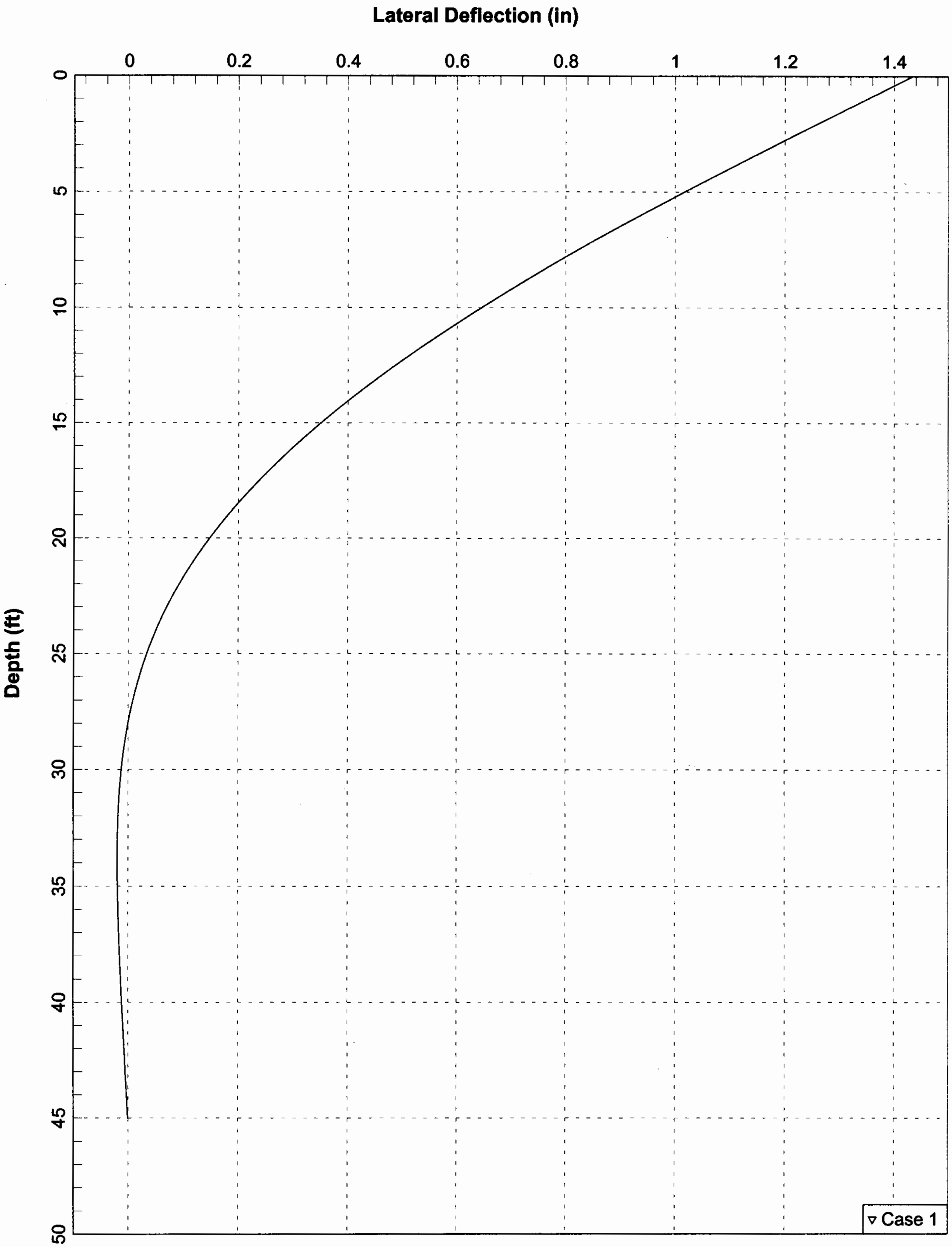
GRI

BORING B-4

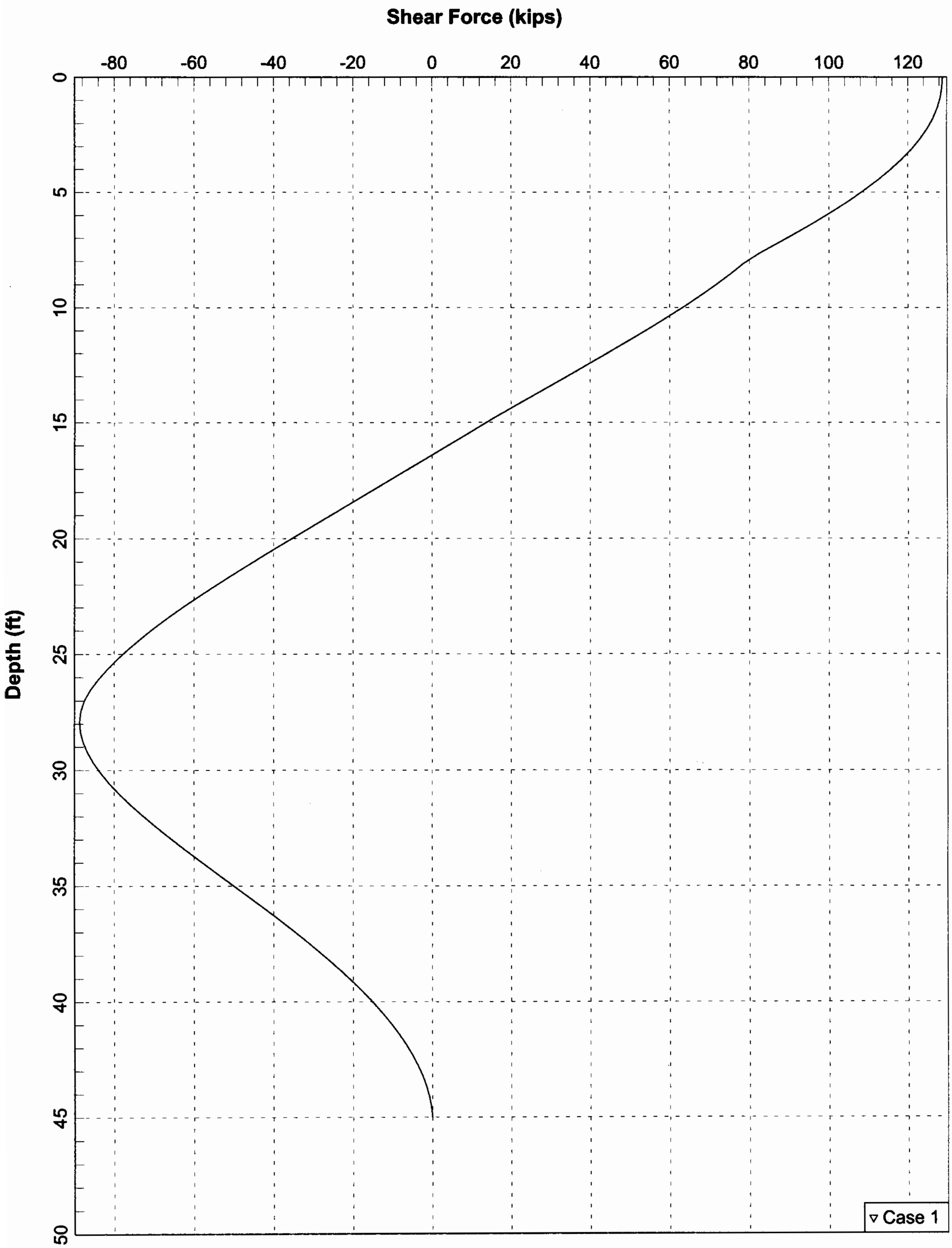
DEC. 2005 JOB. NO. 4317 FIG. 4A

APPENDIX B

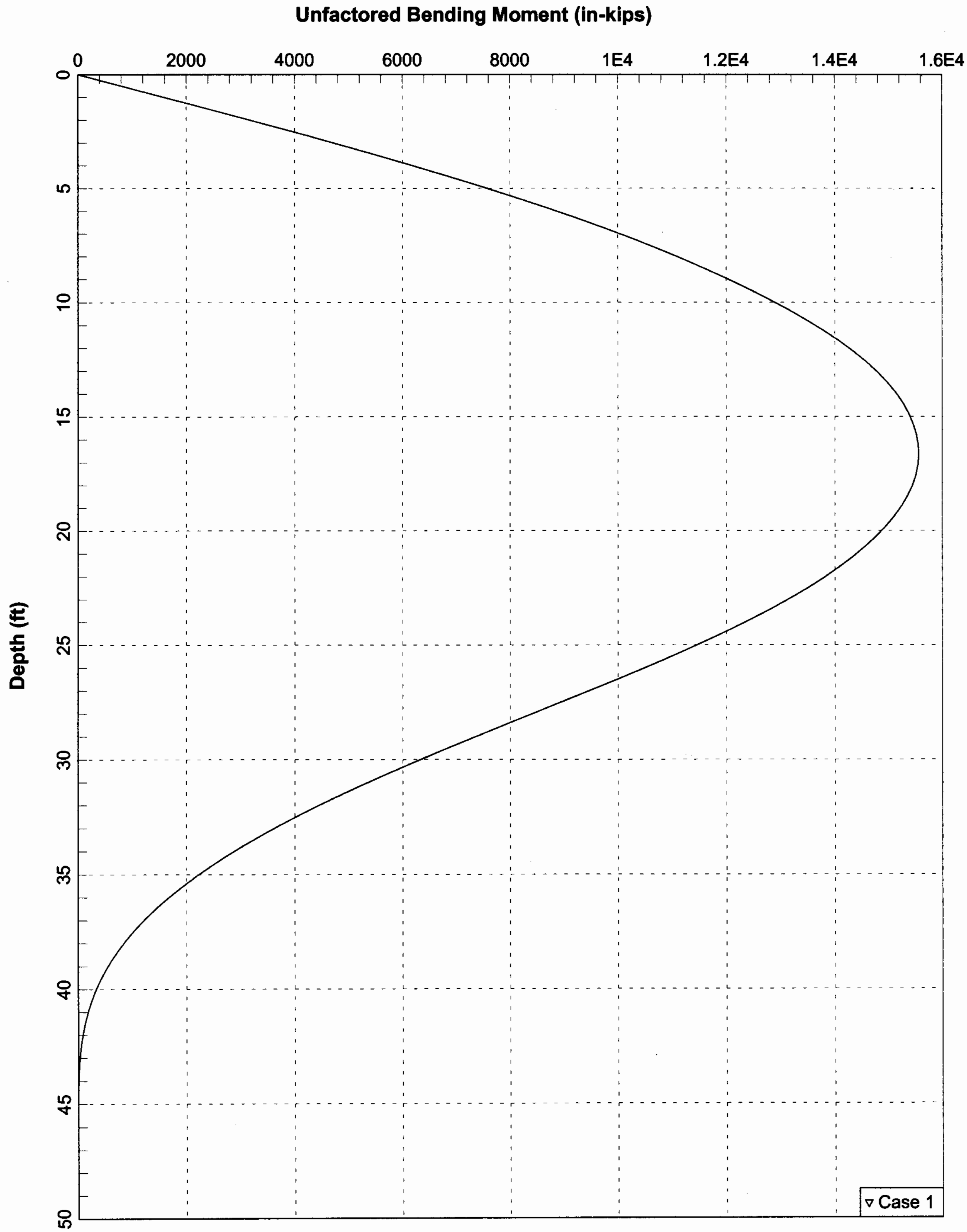
L-Pile Results



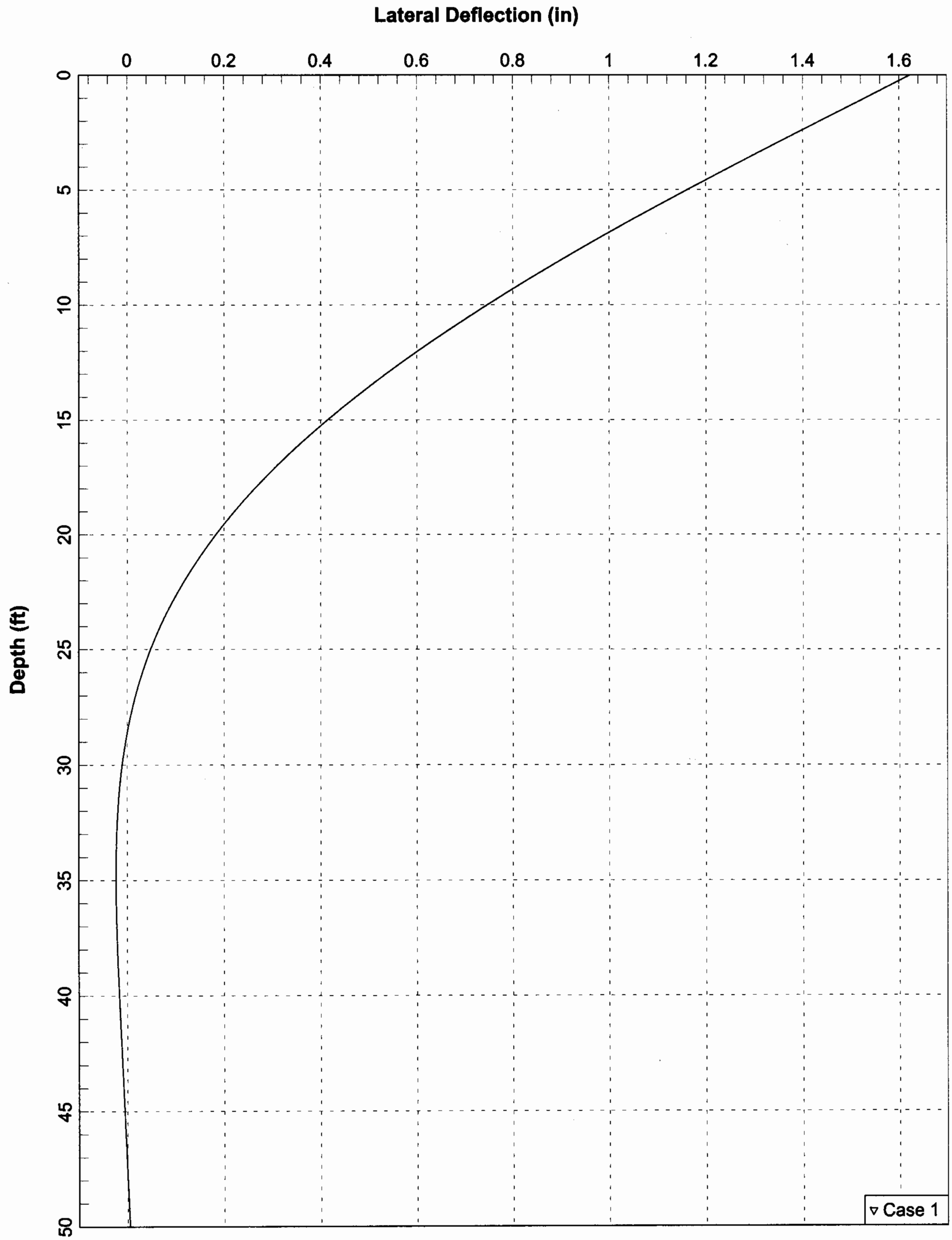
#4317 Klineline Bridge, North Abutment, Long. Load and Pinned Head



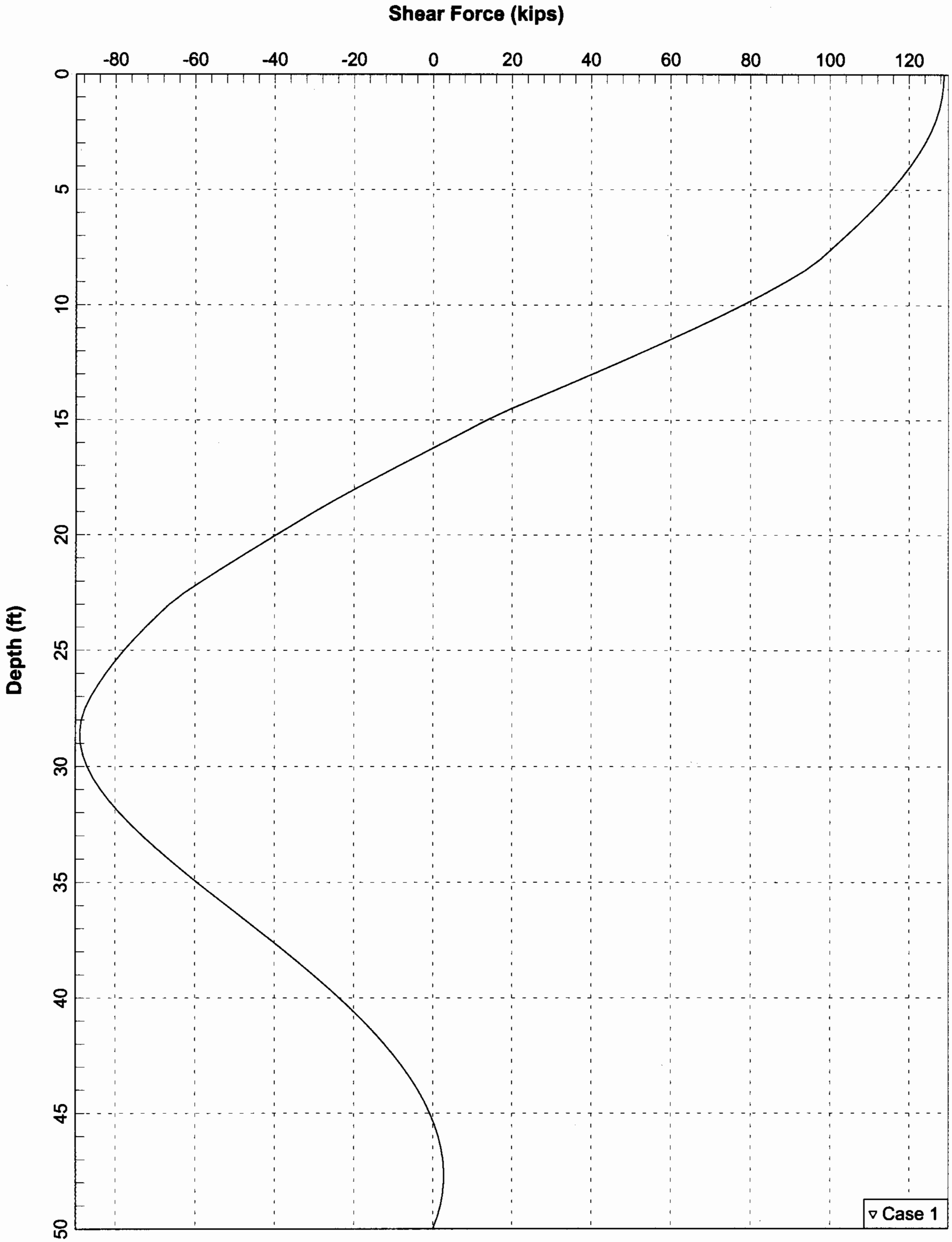
#4317 Klineline Bridge, North Abutment, Long. Load and Pinned Head



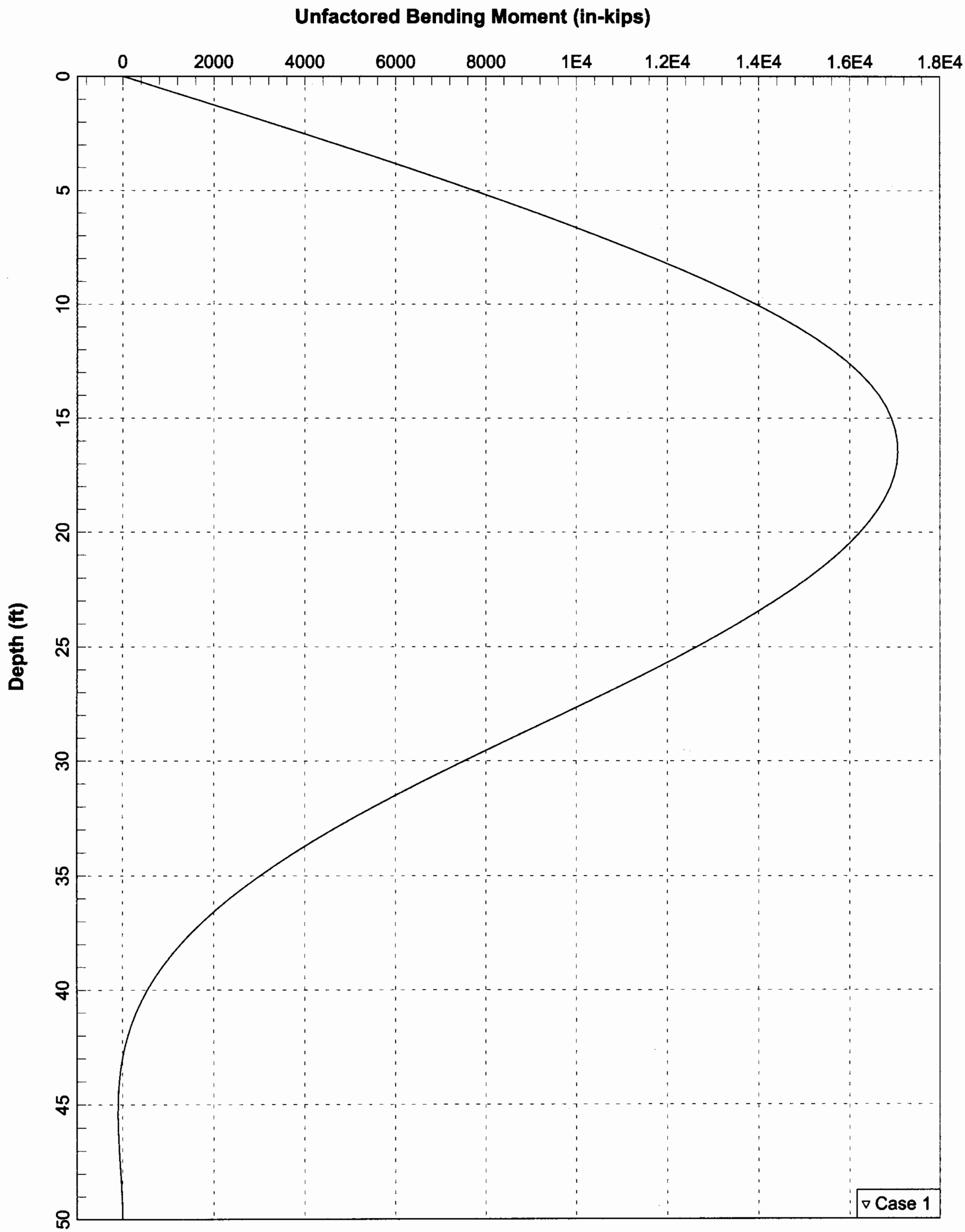
#4317 Klineline Bridge, North Abutment, Long. Load and Pinned Head



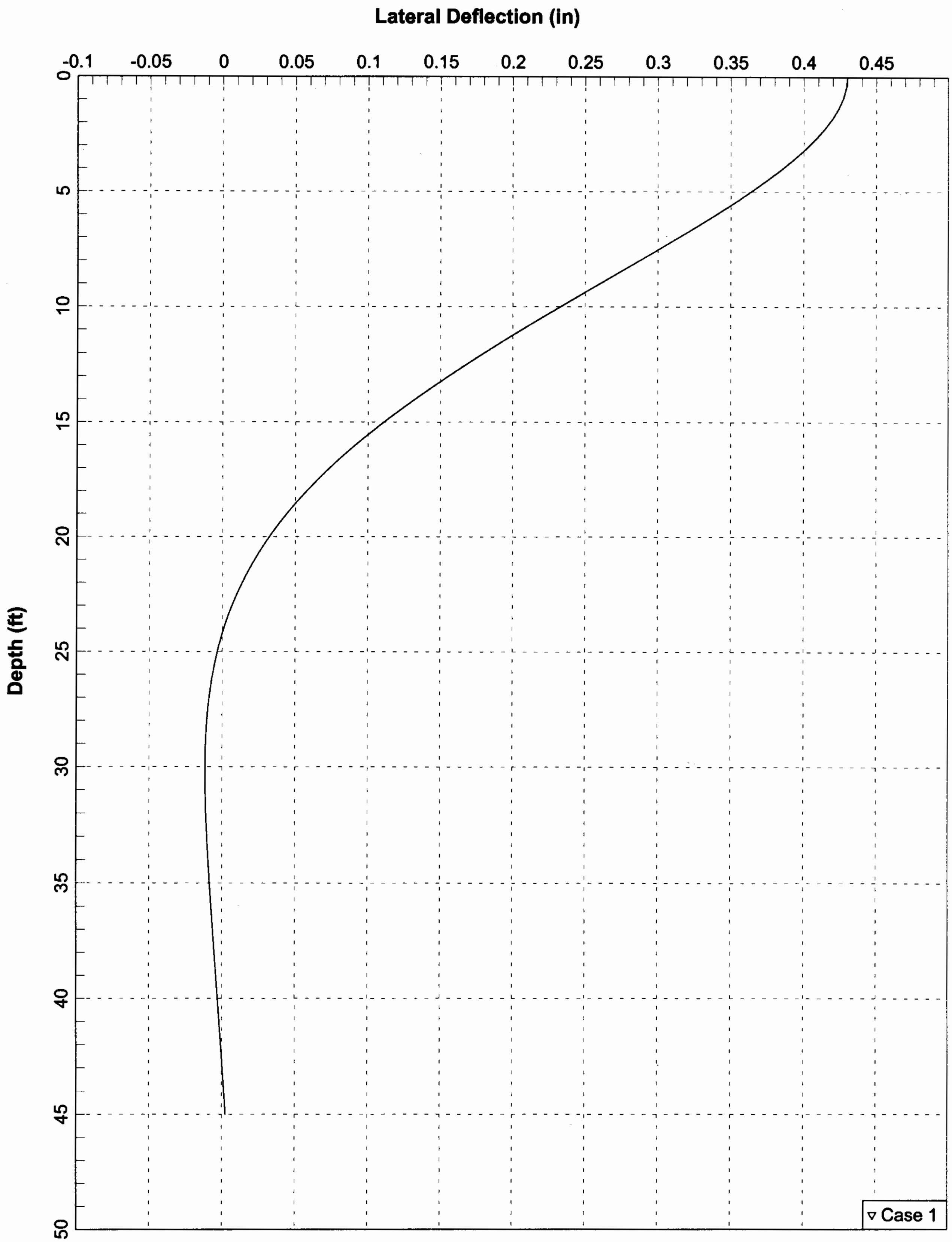
#4317 Klineline Bridge, South Abutment, Long. Load and Pinned Head



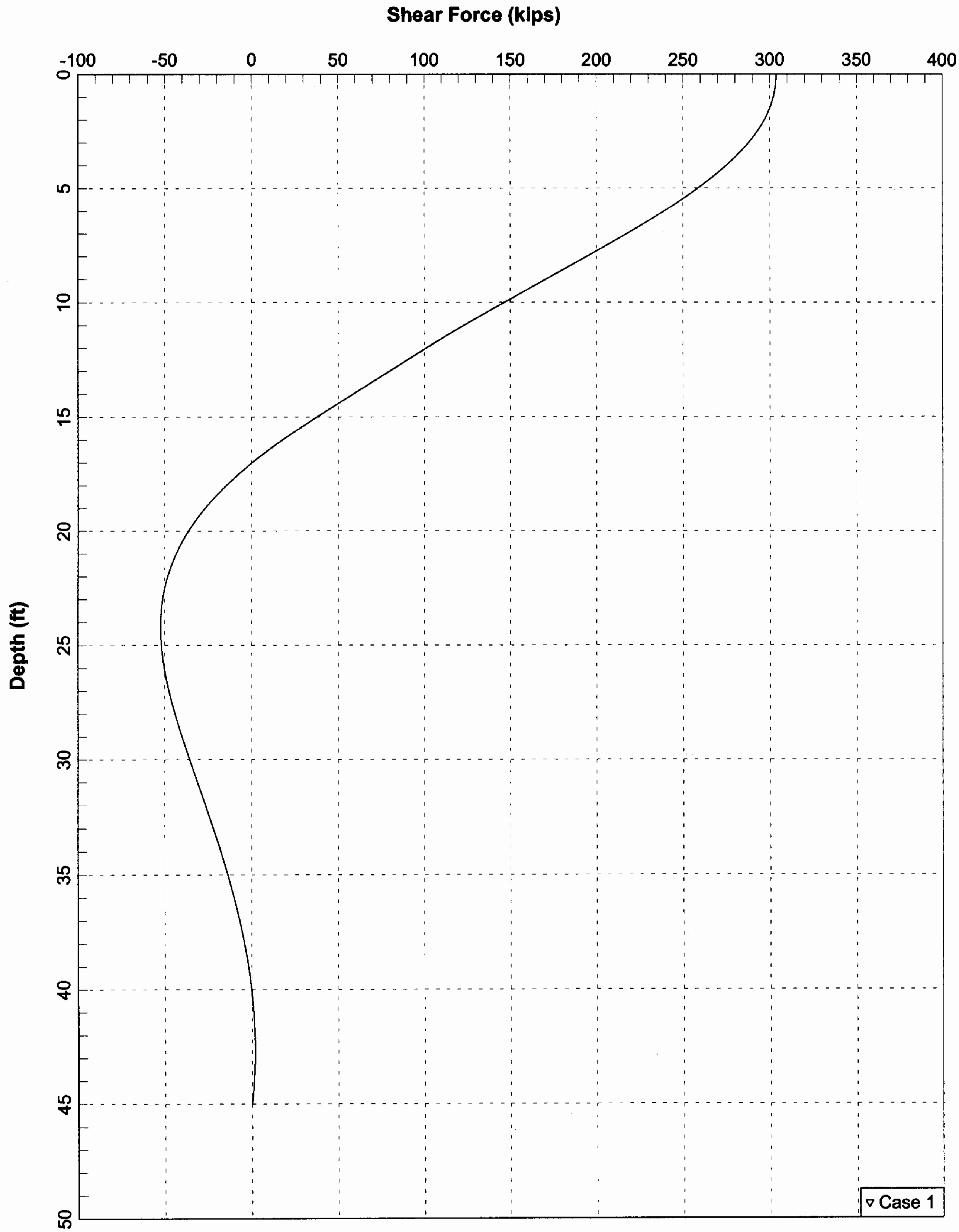
#4317 Klineline Bridge, South Abutment, Long. Load and Pinned Head



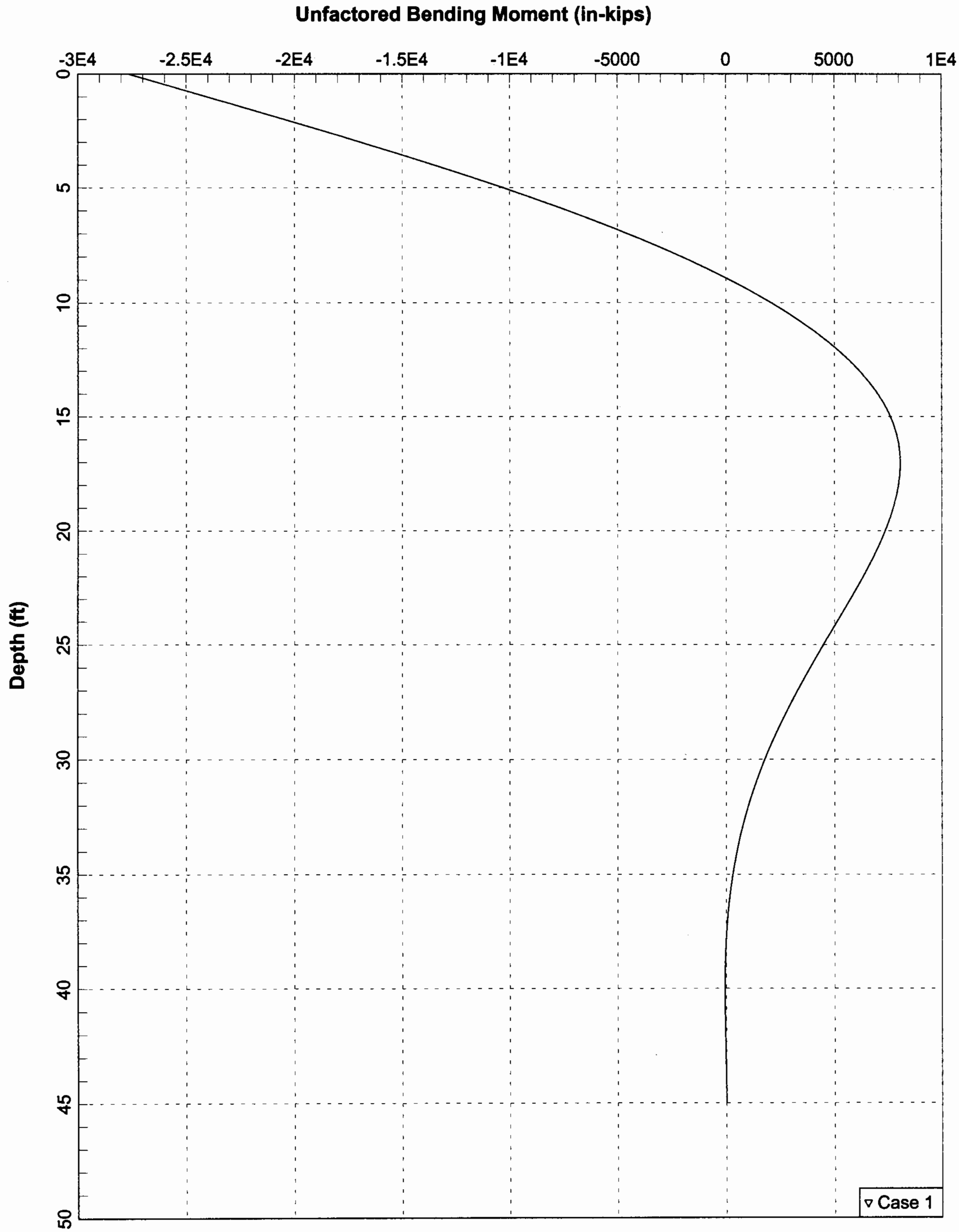
#4317 Klineline Bridge, South Abutment, Long. Load and Pinned Head



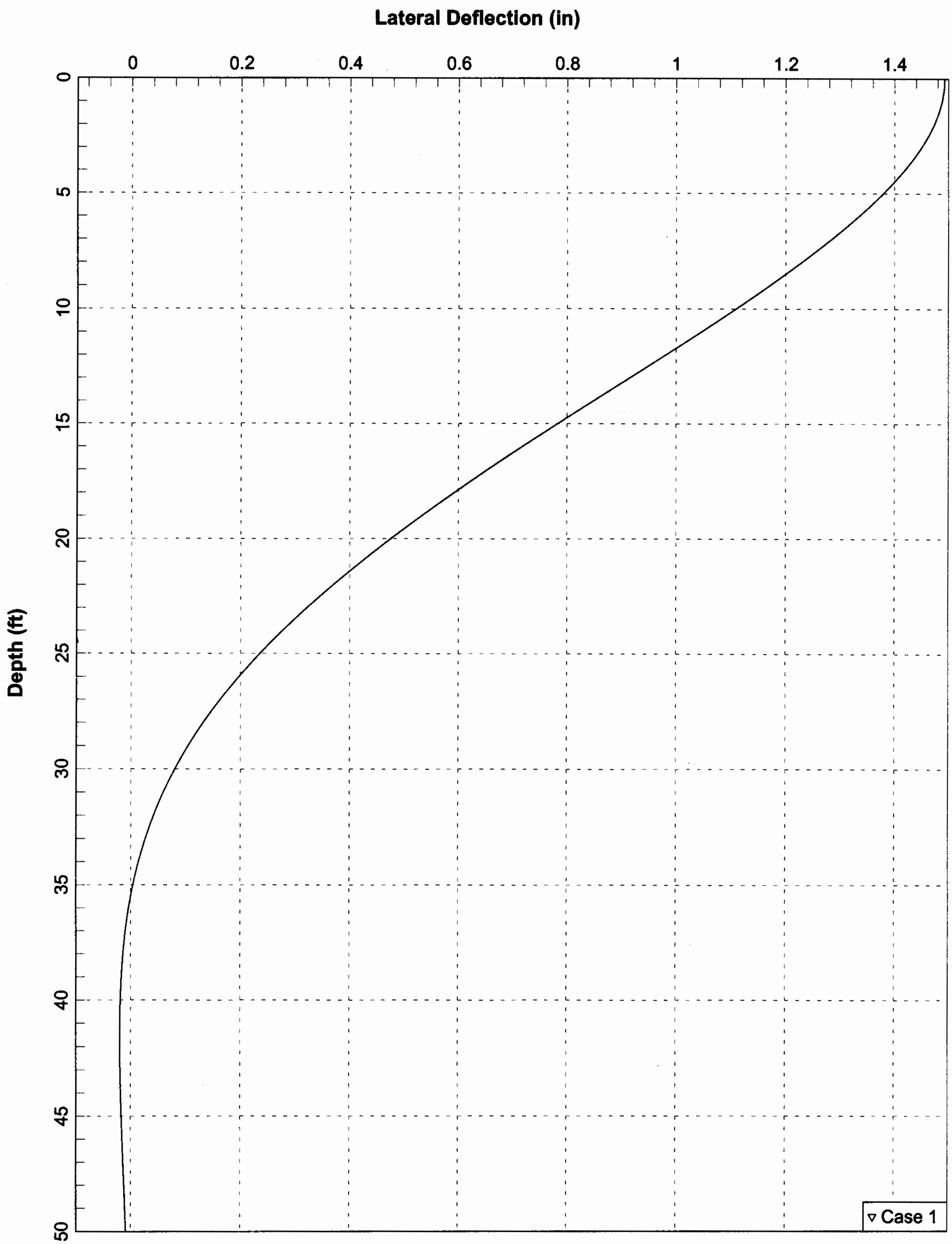
#4317 Klineline Bridge, North Abutment, Transverse Load (303 k) and Fixed Head



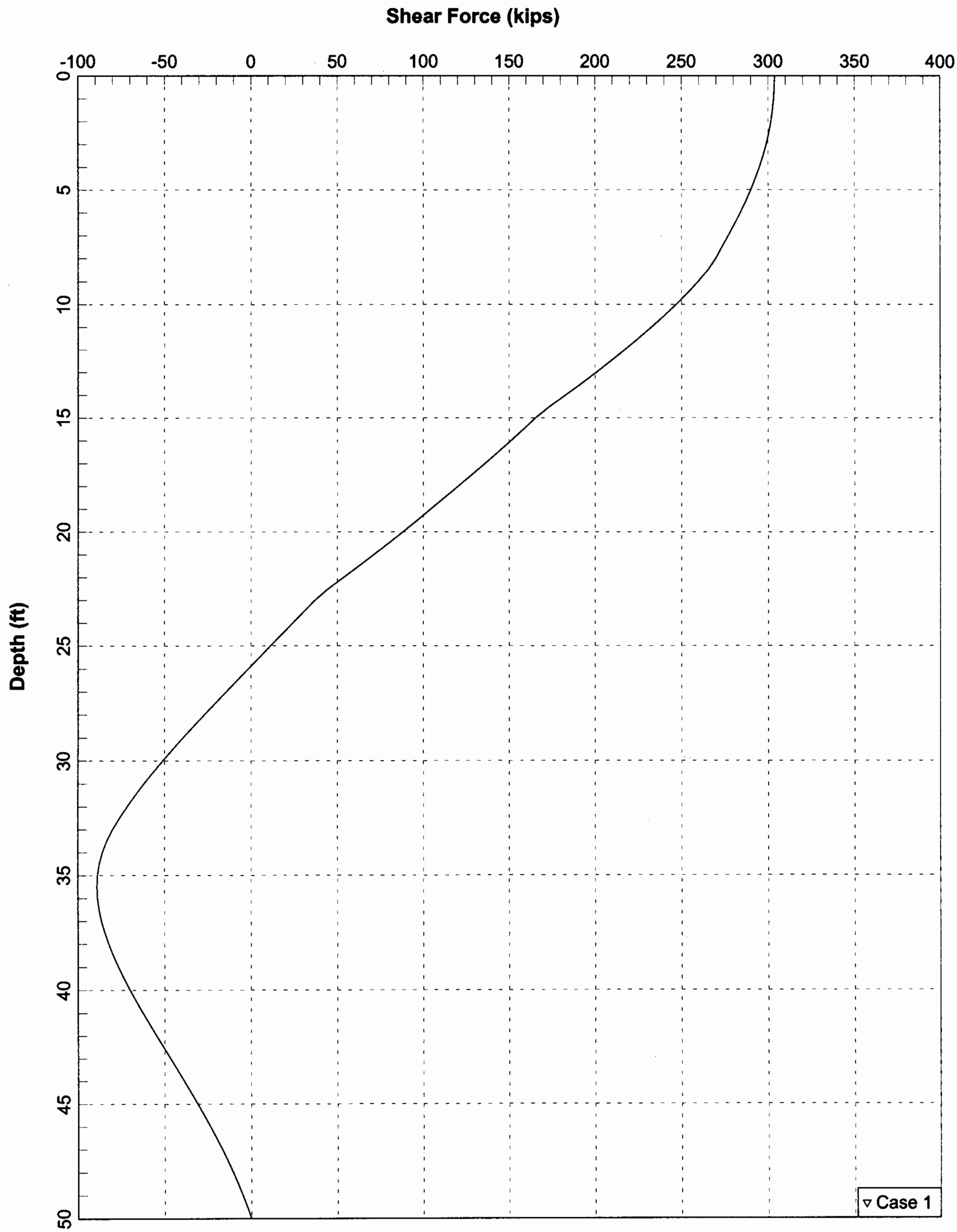
#4317 Klineline Bridge, North Abutment, Transverse Load (303 k) and Fixed Head



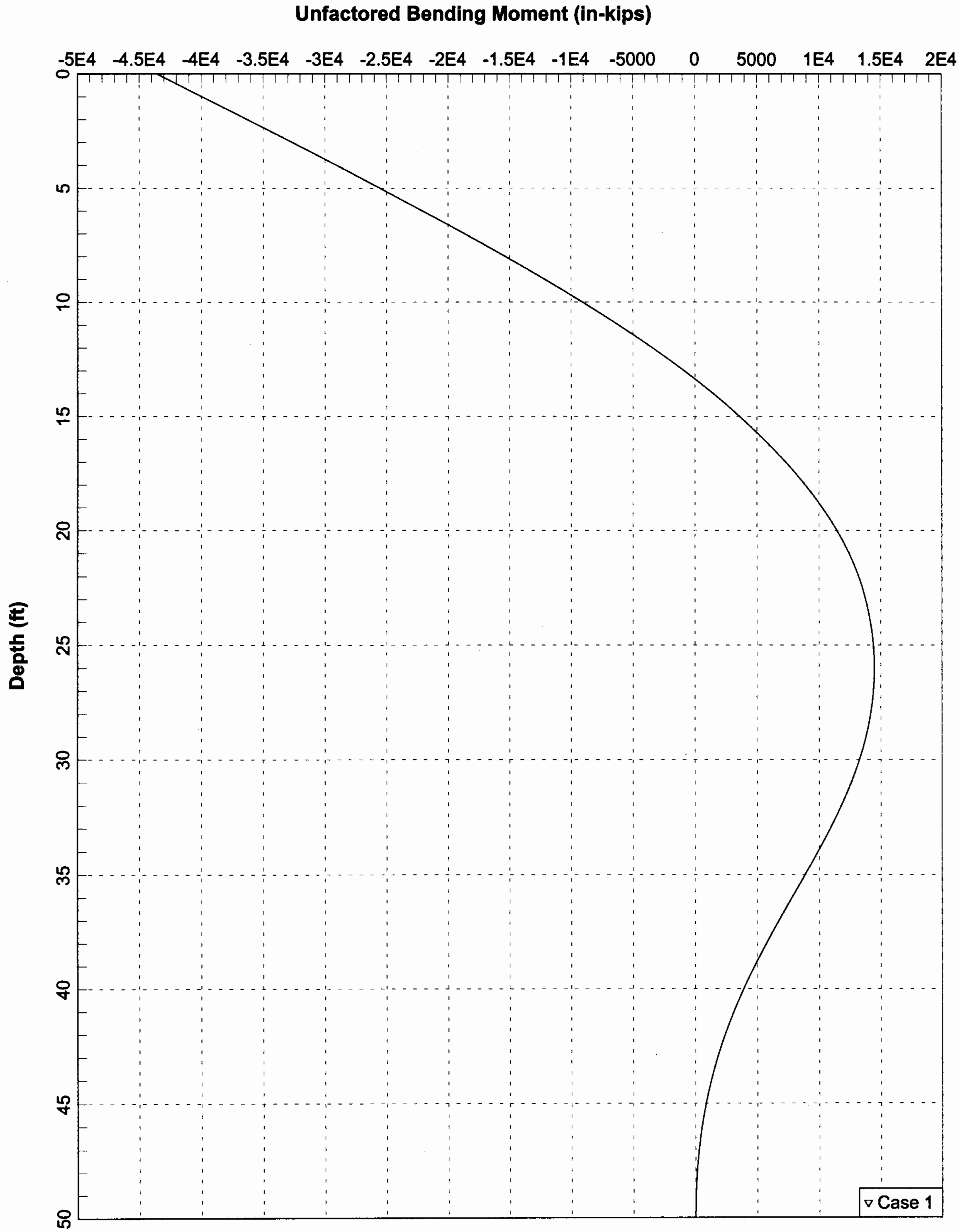
#4317 Klineline Bridge, North Abutment, Transverse Load (303 k) and Fixed Head



#4317 Klineline Bridge, South Abutment, Transverse Load (303 k) and Fixed Head



#4317 Klineline Bridge, South Abutment, Transverse Load (303 k) and Fixed Head



#4317 Klineline Bridge, South Abutment, Transverse Load (303 k) and Fixed Head